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DESIGN AND RESEARCH FOR WELDED STRUCTURES

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STRUCTURAL DIVISION

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DESIGN AND RESEARCH FOR WELDED STRUCTURES

La Motte Grover, M. ASCE

Ever since the pioneering days of welding in steel construction, a number of questions have been asked repeatedly by designers. They are questions that naturally arise in the mind of one who contemplates the use of a type of structure or method of construction which he has not yet encountered in his own experience.

If he is not familiar with welding, he asks, "How can I make sure that the welded joints will be adequate and my structure safe? Must I put special requirements in my specifications for structural steel material to assure weldability? How do welded joints and members behave under repeated stress, impact and vibration? Do residual stresses, caused by welding, reduce the strength of a welded joint or member?"

Although experience during the last 25 years has provided a good answer to many of such questions, it has required extensive research to answer some of them. As in other fields, research is never-ending. It continues to contribute to our better understanding of the behavior of welded structures and how to design them.

These questions have been prompted to some extent by difficulties encountered in a few of the pioneering welded bridge projects in Europe, and more recently in large welded ships built during World War II.

Ship Fractures

Experience and research indicate that ship fracture problems are quite different from those of structural and bridge engineers. Riveted ships, also, have developed many fractures from time to time, and some of them have broken in two.

The wide expanses of relatively thick plating in the hull of a large ocean going ship, constructed by practical methods, present a special problem of stress distribution. Research and study of the ship fracture problem have been centered largely upon the steel material, but it is important to note that improvements have been made in details of design. These improvements have been mainly in the form of eliminating serious notch-efforts or geometrical stress raisers, and eliminating details that did not provide suitable access for making sound welds.

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The very satisfactory record of the Victory ships, built during the latter part of the war, as compared with the earlier built Liberty ships, is most significant. In the Victory ships hatch corners are rounded to reduce the notch effect. These corners and the hatch side girders are arranged so as to eliminate details that make sound welding difficult or impossible, such as were involved in some parts of the Liberty ships. Other important improvements were made in details of design for similar purposes.

Some of the improvement in performance can be ascribed, no doubt, to improved workmanship, as workmen gained experience. Many of them had virtually no mechanical or construction experience or training, at the start of the shipbuilding program.

In large ships built during the last several years, since the war, improvements in details of design and in workmanship have been supplemented by improved hull steel material. No major fractures have occurred in these later ships.

An extensive program of research on large scale specimens and on large welded ships has been carried out to investigate the effect of residual welding stresses. Much information has been developed upon their magnitude and distribution, but no direct evidence has been found to indicate that such residual stresses are an important factor in causing fractures in ships.

Early Welded Bridges in Europe

Although improvements in design have been made in European welded bridges, since their early difficulties were experienced, these difficulties seem to be attributable principally to the grade of steel material that was in use at that time, some 15 years ago—a basic Bessemer steel with bad segregations and subject to serious strain aging; and very different from structural steel material as customarily furnished for bridges and buildings in the United States.

Some rolled beams of this Thomas steel (war surplus material), with butt welded splices, were tested in France shortly after the war. They failed by brittle fracture under static loading, at a stress of only 21,300 psi. By contrast, to fracture some of the welded beams of ASTM-A7 steel, in a current research program at Columbia University, it has required a combination of low temperature and very severe impact, (roughly equivalent to about 70,000 to 90,000 psi static load stress)—much more severe than the impact in actual service conditions for a railway or highway bridge.

Processes Used for Structural Welding

Metal Arc Welding is used almost exclusively for structural work. A very large majority of the welding is done by the manual process.

In structural welding, for the most part, the comparatively short runs of welding are not well suited to fully automatic welding.

For structures with complicated framing, such as the Division Street Interchange of the Bay Shore Freeway in San Francisco, Fig. 1, welding offers a special advantage in simplifying structural connections and facilitating their detailing. Some of the pier bents required the beams or caps to be cantilevered

from one column, and other unusual details, to maintain clearances for roadways beneath. This is one of the largest welded bridge projects ever undertaken.

Although the use of aluminum for building construction has been restricted largely to ornamental uses and details such as window frames and sash, the Navy Department is now making extensive structural uses of the 61ST-6 alloy of aluminum to reduce weight in making alterations in its war ships. A comparatively new semi-automatic welding process is employed, known as the inert gas shielded metal arc welding process, (Fig. 2) using a continuously fed, consumable wire electrode.

The development of this process, together with developments now in progress for new high strength weldable alloys of aluminum, promise to open up new fields for structural uses of aluminum. The present high strength aluminum-magnesium-silicon alloys, such as 61ST, depend upon a solution heat treatment and artificial aging, for their high yield point and tensile strength. Consequently, in the zone affected by the heat of welding, their tensile strength is drastically reduced, even with the newer welding process that has been mentioned, with its more concentrated heat. The new high strength alloys of aluminum now under development contain comparatively high percentages of magnesium as a principal alloy constituent. They are expected to develop very high welded joint efficiencies, especially when welded by the inert gas shielded metal arc welding process.

This process, in both the semi-automatic and fully automatic forms, is used for joining other metals such as bronzes, and stainless steel. It is coming into use also for some types of applications in the welding of mild and structural grades of steel.

Experience and Research as a Background for Design

The last 25 years of experience now provides an excellent background for the use of structural welding for railway bridges, highway bridges and buildings. A good deal of this valuable experience was developed during the war in the tremendous ship-building program where many of the same fundamental problems were encountered.

Outstanding programs of welded bridge construction such as those in the States of Kansas and Connecticut and in the Province of Quebec in Canada, have been especially valuable proving grounds, with their excellent records of service performance over periods up to 16 years, through winter temperatures of -30 deg. F., in Canada. The States of New York and California have now embarked upon most noteworthy programs of welded bridge construction.

In welded building construction, much credit is due Gilbert D. Fish (M. Am. Soc. C.E.) who has contributed a great deal to the advancement of structural welding, since the early pioneering days. In the recent rapid advancement and improvement of design economy for large welded rigid-frame type office building construction in Houston and the Southwest, Boyd S. Meyer (M. Am. Soc. C.E.) has played an important role.

However, this rapid progress would not have been possible without the benefit of a great deal of practical research and testing to destruction of large or full scale welded specimens (Fig. 3) which have been done in programs of the Welding Research Council's Structural Steel Committee, such as the testing of very heavy box girders at the National Bureau of Standards, and

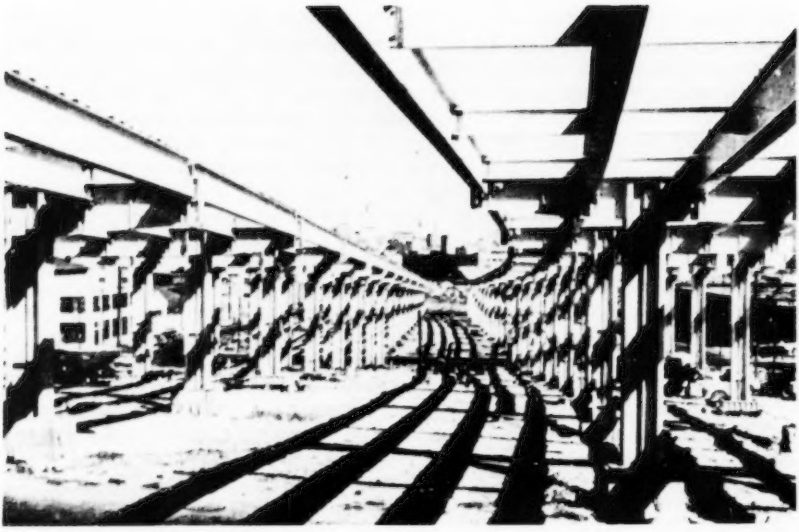


Fig. 1. The all-welded Division Street Interchange of the Bay Shore Freeway, at the west approach to the San Francisco Oakland Bay Bridge.

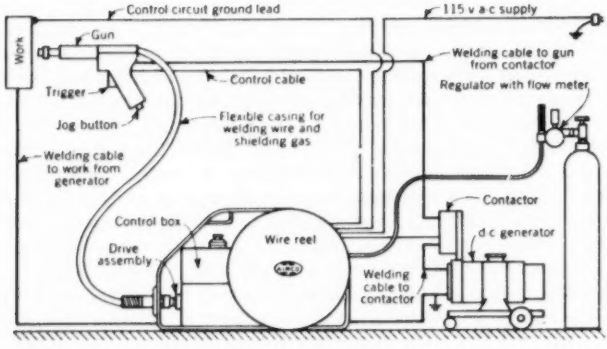


Fig. 2. Schematic Diagram of Inert-Gas-Shielded Metal Arc Welding Process

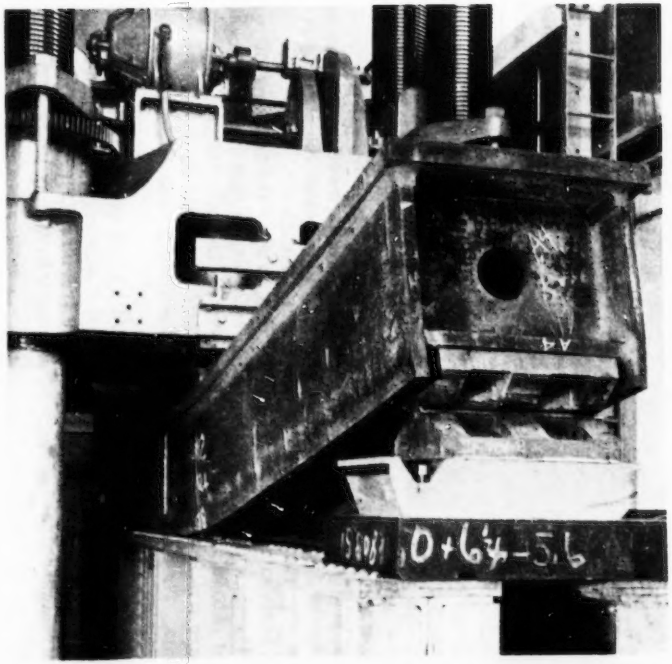


Fig. 3. Testing a 9-ton, 22-foot welded box girder of 1-1/2" and 3" thick material at the National Bureau of Standards.

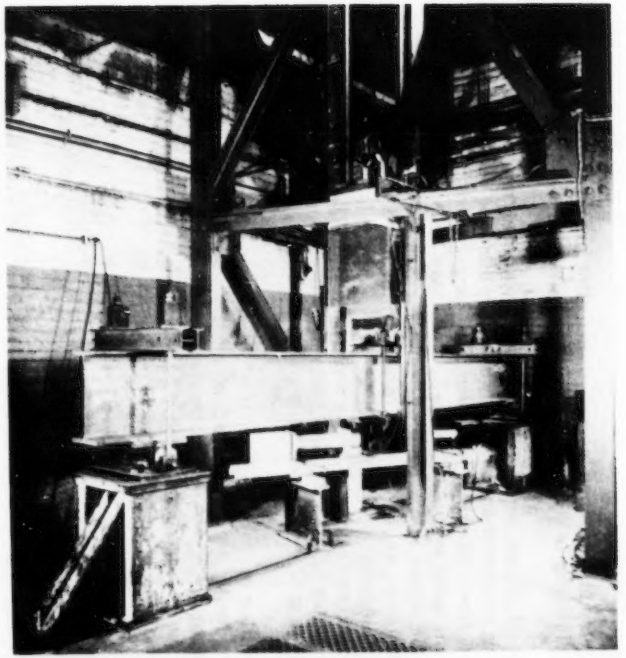


Fig. 4. Impact test of a welded beam at Columbia University.

other projects at various universities, including programs of the Fatigue Testing Committee of the Welding Research Council, at the University of Illinois.

The specifications and codes of the American Welding Society, for bridge and building construction, have drawn heavily upon the results of this research. Design practices, likewise, have been built largely around these results and have become quite well standardized. They have been described in some detail by the writer in a Manual of Design for Arc Welded Steel Structures, and in articles on welded bridges, which are included in the references at the end of this paper.

There are two major programs under way at this time for the Structural Steel Committee of the Welding Research Council, one at Lehigh University and the other at Columbia University. These will be described in some detail.

In a smaller but important project at the Massachusetts Institute of Technology fundamental information on elastic stability has been developed as a basis for specification requirements pertaining to details and proportions of welded members in compression. Some work has been started at the University of Florida on stress distribution in welded cover plates.

Impact Tests of Welded Beams

To determine the best design details and procedure for making a welded splice in a beam or girder, a project of the Welding Research Council was started at Columbia University several years ago, under the direction of Professor W. J. Krefeld, M. ASCE head of the Civil Engineering Research Laboratories, assisted by Dr. E. C. Ingalls, A. M. ASCE until recently. Particularly attractive cost savings had been found for welding in girder construction, but at that time there was a question in the minds of some designers as to whether residual stresses set up by welding might not combine with load stresses and reduce the strength of a welded girder, particularly under suddenly applied loads.

Wide-flange beams, 16 in. W F 71 lbs., of 12-foot span length, with butt welded splices at mid-span, were first tested at room temperature under a load at mid-span. Following static control tests, impact tests were made by dropping a heavy tup on the beam, using a rig somewhat like a pile driver. (Fig. 4).

Residual stresses of considerable magnitude were measured on one beam specimen by the destructive method of relaxation, trepanning plugs upon which SR-4 electric strain gauges had been mounted.

In drop hammer impact tests at room temperature the rolling mill residual stresses, together with the residual welding stresses, and stress concentrations at the point of load application, simply lowered the apparent proportional limit. There was no evidence of adverse effect upon the carrying capacity of the beams. They all failed by lateral buckling or web crippling. (Fig. 5, top).

The program was then redirected toward a study of behavior under impact at low temperatures, and an investigation of transition from the ductile buckling type of failure to failure by brittle fracture. (Fig. 5, bottom). Any ordinary structural steel becomes increasingly brittle as its temperature decreases.

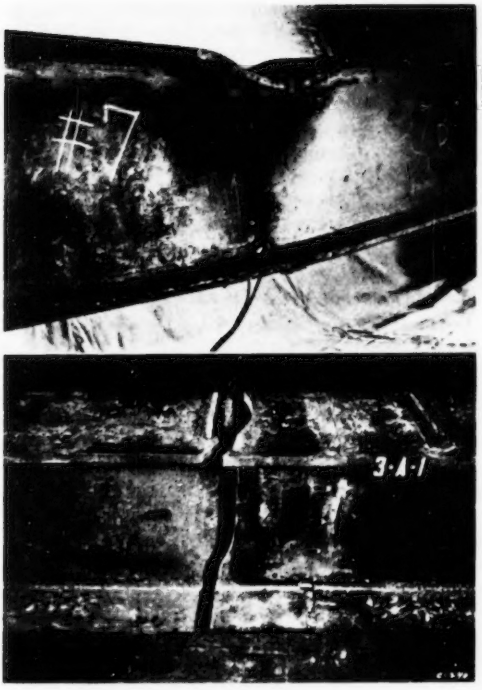


Fig. 5. (Top) Welded beams tested under severe impact at room temperature have all failed by lateral buckling and crippling. (Bottom) Beams tested under low temperature impact failed by brittle fracture—in this case by fracture some distance away from the welded splice.

Beams of two grades of steel material have been tested, both conforming to ASTM-A7. One grade was of semi-killed steel as normally supplied. The other carried a further provision requiring it to be silicon-aluminum fully-killed steel. This steel should not be confused with Structural Silicon Steel ASTM-A94, a higher carbon steel not recommended for welded construction.

Various kinds of small specimen tests were made to compare the two steels with regard to transition temperature from ductile to brittle fracture, using Charpy keyhole and V-notch impact tests and tear tests. For Charpy V-notch specimens taken from similar locations in the beams, the fully-killed steel withstood 40 to 70 degrees lower temperature of testing than the semi-killed steel (based upon an energy level of 15 ft.-lb.) Specimens taken from the flange or web adjacent to the fillets showed up appreciably poorer than specimens from the edges of the flange or center of the web. This may be due to the degree of hot working the material in the steel mill and to different temperatures and rates of cooling during finishing passes in the steel mill. It has been reported that, in general, lower finishing pass temperatures in rolling a steel material are beneficial in controlling its grain size and reducing the temperature at which transition occurs from ductile or high energy fracture to brittle or low energy fracture. Also, in Europe, the flange-to-web junction in a steel beam has been recognized as somewhat of a critical location.

Throughout all of this investigation very careful control has been exercised by the laboratory. Most ingenious and quite elaborate instrumentation has been devised for measuring and recording instantaneous values of rates of strain and deflections. A great deal of valuable fundamental information has been produced on behavior of steel material under impact.

It is recognized that the testing conditions have been much more severe than service conditions for a practical steel structure such as a railway or highway bridge. However, plastic deformation was produced readily in room temperature beam tests when instantaneous rates of deformation in the material were as great as 2 in. per in. per second. This is surely much greater than would be expected in any highway or railway bridge. Also, some of the testing temperatures were of the order of -120 degrees F. Such exaggerated conditions are sometimes necessary to provide complete data on trends in behavior, and to establish correlation between tests of full size structural members of this kind and tests of small specimens of the same material. Correlation of this kind is very badly needed by engineers. Further, such extreme conditions of impact might be produced by collision, bombing or other accident.

Many tests were made at temperatures to be expected for outdoor structures in cold climates, but the severity of the impact and strain rates produced was still exaggerated, to produce actual fractures, (Fig. 5 bottom). However, under static loading no fractures could be produced, even at -100 degrees F.

The criterion established for evaluating the results of the impact tests of these beams was the transition temperature (of testing) when type of failure changed from buckling and crippling to failure by fracture. Beams of both types of steel were tested in several conditions, including: (1) As-welded, (2) Welded and subsequently given a so-called 1150 deg. F. "stress-relief" or post heating, and (3) Welded with 350 to 400 deg. F. preheat to reduce the rate of cooling after welding.

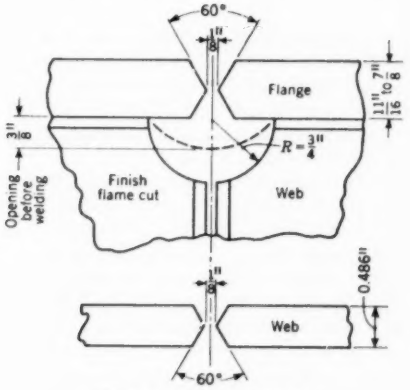
Effect of Low Hydrogen Electrodes, Preheat and Stress Relief

In some of the later tests the benefit derived from the use of electrodes with low hydrogen content in their coatings, such as Class E-6016 and E-7016, has been investigated. The use of such electrodes must not be considered as a panacea that will permit other precautions to be eliminated altogether in the welding of very thick material or some kinds of steel material that are especially difficult to weld. Nevertheless, reports from practical experience indicate a very substantial benefit from their use. One structural fabricator has reported the successful welding of heavy press frames of 5 in. thick material with no difficulties, and with very little preheat when low hydrogen electrodes were used.

A somewhat different technique of welding is used with the low hydrogen electrodes. Operators should be familiar with this technique.

Research has shown that the weld metal, itself, produced by such electrodes (when they are properly conditioned and stored) has substantially greater low-temperature notch toughness than ordinary carbon steel weld metal produced with the common types of electrodes. Impact tests of beams, made since the original presentation of this paper, have shown a very substantial improvement of low temperature impact resistance when such low-hydrogen electrodes are used.

From the beam-tests made, transition temperatures from buckling failure to fracture failure have been fairly well defined for beams welded with ordinary types of mild steel electrodes, Class E-6011 and E-6020. These are given in Table I, along with later data from tests of some beams welded with low hydrogen electrodes, and other beams welded without the provision of the coped holes shown in Fig. 6.



Welding preparation details

Fig. 6. Coped hole in web of beam adjacent to flange, introduced to facilitate making butt weld of flange.

It will be noted that preheating is more beneficial than a stress relief carried out at some time subsequent to cooling after welding. Fortunately, such preheating is a much more practical operation than the so-called

TABLE I

TRANSITION TEMPERATURES OF BEAMS
TESTED UNDER DROP-HAMMER IMPACT

(Based upon temperature at which mode of failure changes from buckling to fracture, except as noted for beams with pressed notches)

	Beam Condition	Fully Killed Steel Beams	Semi-Killed Steel Beams
I	Welded with E6011 and E6020 electrodes.		
	A- As welded	-29 deg. F.	-5 deg. F.
	B- Welded with 350 deg.* preheat	-57	-17
	C- Welded and stress relieved	-45	-5
II	Welded with E7016 (low-hydrogen) electrodes.		
	A- As welded	-65	-25 (or lower)**
	B- Welded with 350 deg.* preheat	-55	-36 (or lower)**
III	With coped web-holes eliminated, and welded with E6011 and E6020 electrodes.	-70	
IV	Plain, unwelded beams with machined coped holes and sharp pressed notch.	+25 (approx)†	+85(approx)†
V	Plain, unwelded beams with coped web holes made by flame cutting	-30	Between -60 and -115‡

* Other research and experience indicate that for beams of this kind of steel and the thickness of material involved, 100 deg. F. preheat, or somewhat less, would be adequate.

** The low-hydrogen electrodes used for the semi-killed steel were not stored or conditioned to control moisture, as they were in case of the fully killed beams of this low-hydrogen series. Somewhat lower transition temperatures for the semi-killed beams might have been found if they, too, had been welded with moisture-controlled low-hydrogen electrodes.

† Since the pressed notches at the coped holes of these beams constituted virtually the initiation of a crack, the criterion for transition temperature is the ability of the steel material to resist propagation of cracks or to inhibit free running cracks.

‡ The number of semi-killed steel beams tested for this condition was insufficient to do more than bracket the transition temperature as indicated.

furnace "stress relieving", when the preheating is of a reasonable temperature such as 150 degrees F. or less. There is evidence from other investigations that a post heat, following immediately after welding, to retard the cooling rate, is at least as beneficial as preheat, but such post heating is not usually very practical. From Table I, it will be noted that for beams of the kind involved in these tests, the use of low hydrogen electrodes is about equivalent to the provision of preheat. Other research and experience have indicated that for this kind of steel and the material thicknesses involved, a preheat of only 100 deg. F., or somewhat less, would have been adequate. However, a 350-deg. preheat was used for these specimens. Semi-killed steel beams welded with low-hydrogen electrodes, behaved as well as fully killed beams welded with ordinary electrodes.

For the fully killed beams welded with low-hydrogen electrodes, no additional benefit was derived from preheating before welding. For the chemistry and thicknesses of steel involved in these beams, the benefits of preheat and the use of low-hydrogen electrodes would be expected to be about equal and not additive.

In view of the severe impact and high strain rates involved in these tests, it is believed that for even the semi-killed steel, welded beams of this kind could be expected to behave in a satisfactory manner at low winter temperatures. For built up girders with very thick flanges, the specifications should include requirements that will definitely insure the provision of fully killed steel material, which would normally be furnished anyway by most mills supplying such very thick material for bridge and building construction ordered to the ASTM-A7 specification.

Effect of Coped Holes in Web of Beam

Semi-circular holes have been coped by flame cutting in the webs of all the welded test beams, (Fig. 6), to provide access for making the butt welds in the flanges. Then the holes have been enlarged slightly to remove any defects at the ends of the butt weld in the web. It was realized that any kind of a hole or geometrical discontinuity causes stress concentrations and raises the transition temperature of a piece of steel. However, this is a practical detail that has been used widely in various countries to facilitate the making of sound flange welds. These small holes are left open because of the difficulty of closing or filling them without leaving defects.

It has been felt that the effect of the hole in the web would not be very serious as compared with holes or defects in extreme fibres such as flanges.

In most of the test beams that fractured at low temperature the fracture in the bottom flange and at top of the coped hole of the web started under the same blow of the tup. It could not be determined whether stress concentrations at the coped hole were an important factor, but very high local plastic deformations were found at these holes, at temperatures approaching the transition range. In a few beams it could be ascertained definitely that fracture started at the coped hole, prior to the blow of the tup that caused extensive fracturing in both web and flange.

Further tests of unwelded beams were made to investigate independently the effect of such flame-cut copes and the condition of their edges. Tests were made of three such beams of fully-killed steel and three of semi-killed steel. The fully-killed beams showed a transition temperature of about 30 degrees F, practically the same as for the as-welded beams of Table I.

For some reason the semi-killed beams showed up much better than the fully killed ones and their transition temperature was below -60 degrees F. However, the tests are too limited to warrant the drawing of conclusions in comparing the effect upon the two steels.

It must be remembered that all these tests represent exaggerated conditions of impact and high strain rate; also that when flame cutting is used in preparing beveled edges for welding, any effects of flame cutting are removed by the local melting and fusion of the welding process.

In other unwelded beam-specimens, with such holes made by drilling and carefully filing the edge to a smooth surface, no cracks developed at the lowest test temperature of -115 degrees F. This led to the belief that surface condition of the edge of the hole was the important factor. It was thought to be a geometrical effect, rather than metallurgical (although there is evidence from research that a suitable flame treatment will increase the resistance to propagation of cracks from such small surface defects).

This belief seems to have been confirmed by further recent impact tests of three unwelded beams of each type of steel, which were prepared with "crack starters" in the form of drilled and mechanically notched holes in the web. A fine notch, 0.10 in. deep of 45 degree angle and root radius of .00075 in. was pressed at the top edge of the lower hole, after first filing the notch almost to size. This technique, which has been used by several investigators recently, produces a sharp notch without appreciable cold working that might influence results. All six of these beams were tested under virtually identical conditions except for varying the test temperature.

Brittle failure occurred in all of these beams with pressed notches when tested at temperatures of -30 degrees and -60 degrees. A semi-killed beam tested at +30° F. likewise failed by brittle fracture, but a fully-killed beam tested at +30° F. failed by buckling. Only one beam, one of the fully-killed steel, was tested at a higher temperature, +60 degrees F. It developed only a very short crack of fibrous texture, 1/16" in length and then continued to buckle without further propagation of the crack.

The test results indicate that the tendency for crack propagation is temperature dependent. It appears that in the case of a brittle fracture, at relatively low temperature, a very large majority of the total absorbed energy is utilized to start the fracture, and a comparatively small portion to propagate the crack. In a ductile fracture at comparatively high temperature, these proportions seem to be approximately reversed, with the greater portion of the energy being utilized for crack progagation.

Since a sizeable brittle fracture or crack was developed in the fully-killed beam tested at -30 degrees F, this unwelded beam with a pressed notch might be considered as having behaved very much as does a beam of the same material in the as-welded condition, or one with a flame cut hole.

These tests demonstrate a fact that has been found true likewise in fatigue testing. Any practical method of fabricating and joining steel material usually introduces at least minor geometrical notch effects that reduce the resistance of the material, particularly under dynamic loading.

The latest tests, which will be reported soon in detail by the investigators, provide some very interesting information on the resistance to propagation of cracks that is afforded by the two different grades of steel, and how this property is related to transition temperatures indicated by Charpy tests. A previous comprehensive report by the investigators is listed in the references at the end of this paper.

To investigate the benefit of eliminating the coped holes in the webs of the beams, adjacent to the top and bottom flange welds, a series of six beams without such copes were fabricated and tested, subsequent to the original presentation of this paper. Since the supply of semi-killed beams had been exhausted, these tests and their comparisons were limited to fully killed steel material. To provide comparison with a larger number of previous tests, the butt welded splices of these test beams were made with E6011 and E6020 electrodes, rather than low hydrogen electrodes.

Although the execution of sound flange welds is more difficult when the copes are not provided, and special care and procedures are required, the precautions required are considered to be entirely practical.

As indicated in Table I, these beams without coped holes showed a transition temperature, from failure by buckling to failure by brittle fracture, of about -70 deg. F., although sectioning the beams for examination did show some very appreciable weld defects at the flange-web junction. This transition temperature is about 40 deg. lower than for similar beams with web holes.

There is a further advantage of employing test conditions more severe than likely to be encountered in practice as involved in this program, when the exaggerated condition is not complicated and can be controlled and evaluated. It serves to magnify the effects of details of design so they can be studied critically. Whenever difficulties are encountered in any kind of engineering construction they usually result from a combination of factors or faulty details and practices. The best practical details of design necessarily involve some notch effects, but a well advised designer can preclude the introduction of especially unfavorable details.

Plastic Analysis in Structural Design

There are especially great advantages for the use of welding in developing highly efficient and economical connections in continuous or rigid frame construction. Consequently, as welding has come into more extensive use for building construction, a great deal of interest has arisen among designers and researchers in the method of plastic analysis of continuous frames and their components. This method has been called by various names such as "Load Limit Design", "Plastic Design" and "Load Factor Design."

The objective in using this method is to apply a suitable safety factor to the true ultimate load that a structure will carry without excessive deformations or deflections, and to produce a design that is as nearly balanced as practical among the component members and connections of a continuous frame. In this way it is possible to attain maximum economy of material and construction costs.

Conventional methods of analysis, based solely upon elastic behavior, usually presume that failure is imminent when a yield point stress is developed at any point in a frame. A critical examination of experience in practical engineering structure shows that this is far from true in most rigid frame structures, and that continuous frames possess a great deal of unrecognized reserve strength.

This general idea is not new. The concept of "plastic hinges", which cause a favorable redistribution of moments and stresses, was introduced at least 25 years ago in Germany, and has been discussed extensively in the

technical press in Belgium, France and England. For many years in this country, the reserve strength accompanying continuity has been recognized to some extent by allowing increased working stresses for combined gravity loads and lateral forces such as wind forces. Further, in any practical engineering structures, very sizeable plastic deformations must actually occur, as the members and connections (whether welded or riveted) become adjusted while initial loads are being received.

Interest in this method was revived several years ago, largely by studies started under the direction of Professor J. F. Baker of Cambridge University. They showed that in some special cases the actual failure load of a continuous frame may be eight or ten times that computed by conventional analysis, and in many cases of continuous framing, very attractive cost savings can be made by extending the design analysis to take into account the reserve of strength in the plastic range. Localized abrupt stress peaks, such as those indicated by elastic analysis at points of negative moment over supports, are not as significant as one might think. Even in columns with rigid connections of beams, there is often a great reserve of strength after yield point stress has occurred.

To be conservative the method of analysis may be devised in such a way that under the assumed working loads for a structure, the computed stresses would not exceed the yield point stress. However, the true ultimate load capacity would be calculated, as limited by excessive deformations or deflections, recognizing the reserve strength after plastic yielding has begun. Thus, in effect, increased working stresses would result when a given factor of safety is applied.

The opportunity to develop a complete rational method of plastic analysis has been enhanced in this country by the provisions that could be made for large scale tests to determine the basic behavior of beams, columns, welded joints and frames in both the elastic and plastic ranges. Most of the research in this country for this method of design has been carried out at Lehigh University under the direction of Professor Bruce Johnston, M. ASCE (now at the University of Michigan) and Professor Lynn S. Beedle, M. ASCE (presently in charge of this work). The program is under the general direction of an advisory subcommittee of the Structural Steel Committee of the Welding Research Council. Funds for the work are being furnished by the Navy Department, the Welding Research Council, the A.I.S.C., the A.I.S.I., Column Research Council, Institute of Research and Lehigh University.

In a paper presented at the 1952 Annual Meeting of the A.I.S.C., Mr. Beedle cites the examples shown in Fig. 7 as a normal case in which plastic design of continuous framing, Case III, shows a saving of 20 percent as compared with Case II designed by the elastic theory; and it provides the same actual load capacity as Case I, designed for simple supports. There is a 40 percent saving as compared with Case I. In a structure that contains some rigid frames and some simply supported beams all designed by the elastic theory, (Case I and Case II), the design as a whole is unbalanced. The beam of Case II is obviously overdesigned.

The basis for this plastic theory of design, as applied to the beam of Fig. 8, is illustrated in an elementary manner by the simplified load-deflection curve and stress distribution diagrams of Fig. 9. Slightly below Point 1 of Fig. 9, yield point stress has barely been reached at the point of maximum moment at the left support. The distribution of stress across the beam section is still triangular, and the resisting moment of the section at start of yielding is designated as My. Slightly above Point 1, some plastic

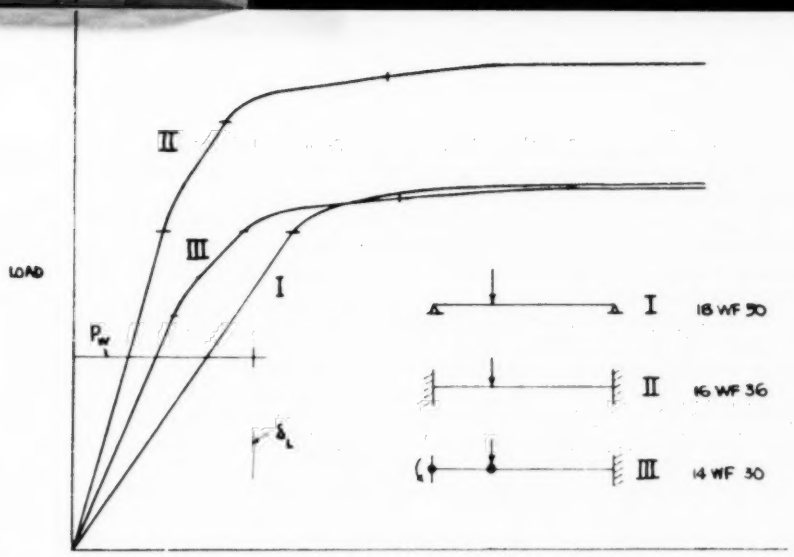


Fig. 7. Load-deflection diagram for three beams of same span length.
I - Designed as simply supported.
II - Designed by elastic theory - fixed ends.
III - Designed by plastic theory - fixed ends.
P_W = working load
δ_L = maximum allowable deflection
Note: Beam diagram for Case III depicts its condition when first plastic hinge has formed at left support and a second hinge is starting to form under the load. When third plastic hinge forms, at right support, the true load capacity of the beam will have been reached.

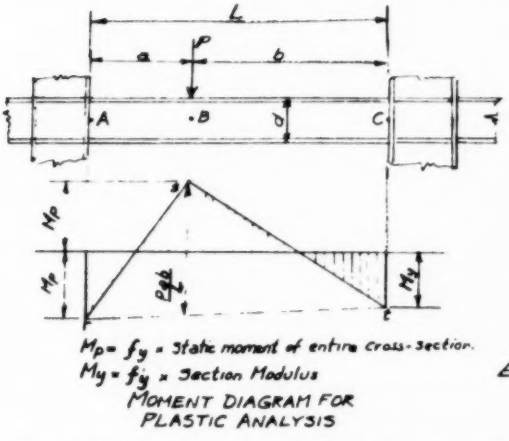


Fig. 8. Moment diagram used for plastic analysis depicts maximum beam moments when third, final plastic hinge forms. Stiffnesses of adjacent members at ends of beam are assumed in this case to be great enough to cause the development of plastic hinges in beam A-C.

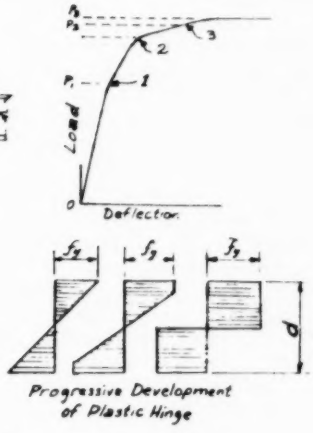


Fig. 9. Progressive development of plastic hinges in beam of Fig. 8.

deformation has started in the outer fibres and the stress distribution becomes trapezoidal. A plastic hinge then proceeds to develop at this point of maximum moment - at the left support in the case illustrated.

For all practical purposes, a rectangular stress distribution can be assumed at the plastic hinge. The resisting moment, Mp, is simply the yield point stress multiplied by the static moment of the entire beam section about its neutral axis.

As the load on the beam is increased further, the beam behaves as though it were hinged at the left support, with a constant moment applied at that point, until the process of forming a plastic hinge has occurred under the load (Point B of Fig. 8). This occurs in the vicinity of Point 2 on the load-deflection curve. At Point 3 of this curve, a final hinge starts to form at the right support. From here on deflection continues at an uncontrolled rate. Therefore, this represents the true load capacity of the beam.

If the rotation at any plastic hinge actually proceeds far enough, the stresses in the beam fibres will start to increase again as strain hardening becomes appreciable, but this does not occur until the strain or local degree of curvature is about 15 times the value at the elastic limit.

In the case of a symmetrical simple portal frame of two similar legs rigidly connected to a beam, with symmetrical loading and no side sway, the first hinge might develop at mid-span. Then, as the load is increased, the moments and stresses at the knees would increase until two more plastic hinges were developed. Collapse would not start until virtually full plastic hinge action had developed at all three of these hinges, when deflections would increase at an uncontrolled rate.

In the simple case of Fig. 8 and 9, only the stresses and rotations caused by bending moment have been considered. In practical design there are sometimes other limiting factors that must be considered. The rotation capacity of each section must be evaluated, to determine that all the assumed plastic hinges can actually be developed before local buckling, excessive deflection or fracture occurs. There are other limitations similar to those that must be investigated in design by the elastic theory. The effect of shear deformations may be important. For example, in rigid frame rectangular knees formed by the intersection of rolled beam sections (without haunching) there is no wide-flange shape that has sufficient web thickness to prevent premature web yielding due to shear, without added stiffening. Expressions have been developed for the required thickness of web in simple rigid frame connections, which apply in the case of elastic analysis as well as plastic analysis. (See references at end of this paper.)

An analysis of a case such as that illustrated in Fig. 7, made by Yang, Beedle and Johnston of Lehigh, showed the maximum deflection of the beam designed by plastic analysis to be less than that of a beam designed for a simple supported condition. However, deflections might be a limiting factor under some circumstances.

For investigating such factors as rotation capacity and web yielding due to shear, there is a special advantage for large scale tests like those being made at Lehigh University, involving continuous beams, columns, rigid frame knees and complete portal frames, fabricated from commercial rolled steel sections such as are actually used in practical construction.

A good many tests of this kind have been made to develop information for the reliable prediction of moment-rotation characteristics.

A special "rotation indicator" device has been developed to facilitate the measuring of the local curvature at any point, so that so-called M/ϕ (moment-curvature) relationships can be determined.

Tests of Rigid Frame Knees

Typical results of tests of various types of rigid frame knees are shown in Fig. 10. Bending moments are plotted against ϕ-values or unit rotations in radians per inch. This diagram also includes an M/ϕ curve for the plain beam section, determined by beam tests made prior to the testing of the knees.

These results show that straight knees with parallel flanges, can be proportioned easily and economically to develop capacities for moment and rotation considerably in excess of those required to meet the design predictions. Knees with curved haunches showed excellent strength when proportioned according to procedures recommended by the American Institute of Steel Construction.

For use in computing the distribution of moments in rigid frame members, approximate expressions have been developed for determining the average unit rotations of haunched connections as compared with simple rectangular knees.

All the sections tested eventually failed by local and lateral instability rather than by fracture. The larger sections collapsed more suddenly. It would appear that if haunched knees were used in plastic design, they should be proportioned strong enough to cause the plastic hinges to develop in the adjacent members. Additional tests of complete frames would be desirable to determine the suitability of such an arrangement.

Of course, information developed in the elastic range is useful in the design of rigid frames even though plastic analysis is not used. Design practices have always been predicated upon the use of a steel material that can deform plastically.

The simple type A connection, (Fig. 11), of lowest fabrication cost, showed less rotation capacity in the plastic range than rectangular knees such as L and M (Fig. 12) with more stiffening; and type A collapsed more suddenly after reaching maximum load capacity. However, the type A connection appears to behave very well within the elastic range and might possibly be used for some cases in plastic design where only limited rotation capacity is needed.

The tests demonstrated the importance of adequate lateral support for any rigid frame knee, for example, at the mid-point of the knee as well as at splices between built-up haunches and adjacent beam sections.

Connection "P" (Fig. 13), without a diagonal stiffener, deformed due to excessive web shear, at half the moment value for the lighter of the two members joined. Although rotation was very large at collapse, involving a good deal of strain hardening, the moment capacity never reached that of the lighter member, and approached that capacity only at very large rotations.

All of the tests of rigid frame knees were made on specimens as welded, that is, without any subsequent treatment such as stress relief.

Other Tests for Plastic Design

The entire program at Lehigh University has included tests of columns with combined bending and axial loading to simulate conditions in the vertical members of rigid frames; also tests of continuous beams, to determine their capacities for rotation, to form plastic hinges. Three rigid frames, portal

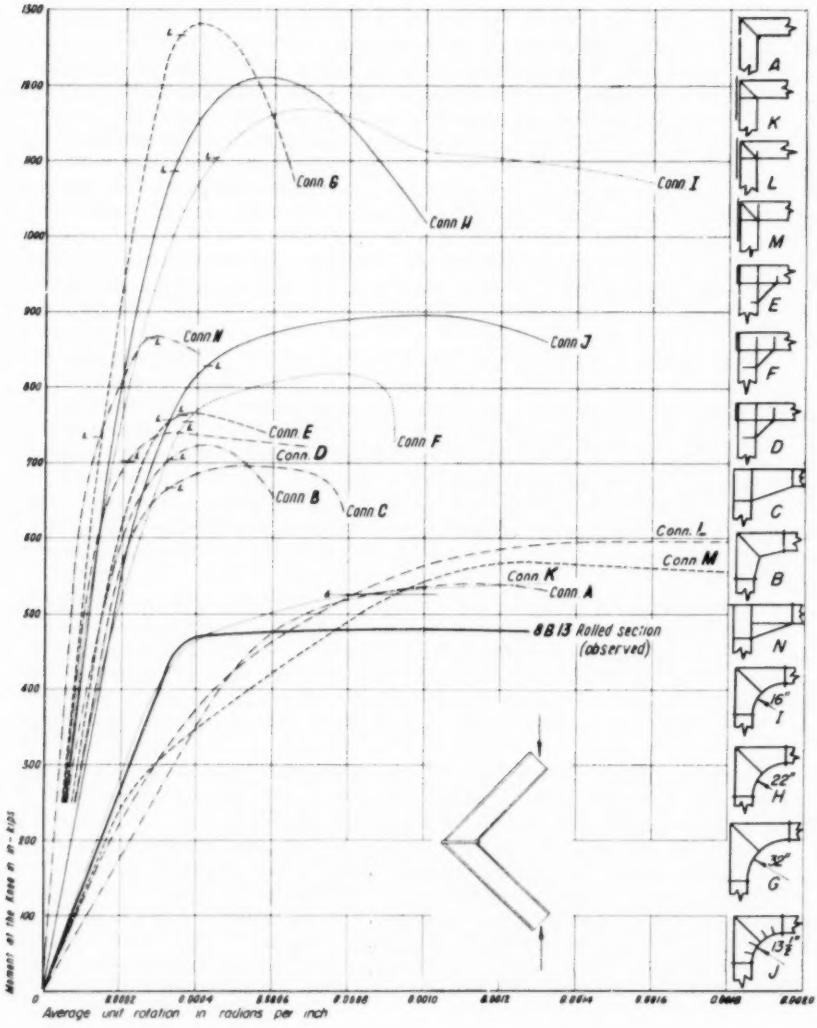


Fig. 10. Moment-rotation diagrams (M-ϕ curves) for various types of rigid frame knees and for 8B13 beam section.

Fig. 11. Type A knee behaved well, especially within elastic range, but did not have as much rotation capacity as rectangular knees L and M.

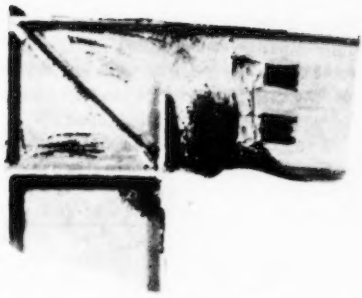


Fig. 12. Type L knee (shown here) and likewise Type M, showed excellent strength and rotation capacity in the plastic range, and did not collapse as suddenly after reaching maximum load capacity.

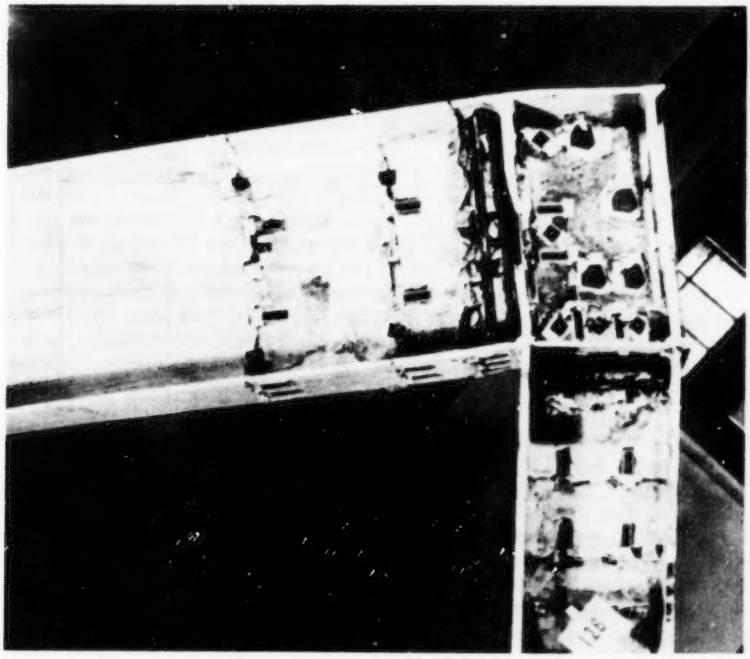


Fig. 13. Type P knee (not shown in Fig. 10) without a diagonal stiffener, deformed excessively at only half the moment value for the lighter of the two sections joined.

frames with rectangular knees, have been tested virtually to destruction. The effect of residual stresses due to welding, as well as those due to cooling after rolling in the steel mill, have been investigated in columns and beams. Most of these tests have been reported in various Research Supplements of the Welding Journal of the American Welding Society, as listed in the references at the end of this paper.

The portal frames tested were of 14-foot beam span between the vertical legs, with two symmetrically located concentrated loads, (Fig. 14).

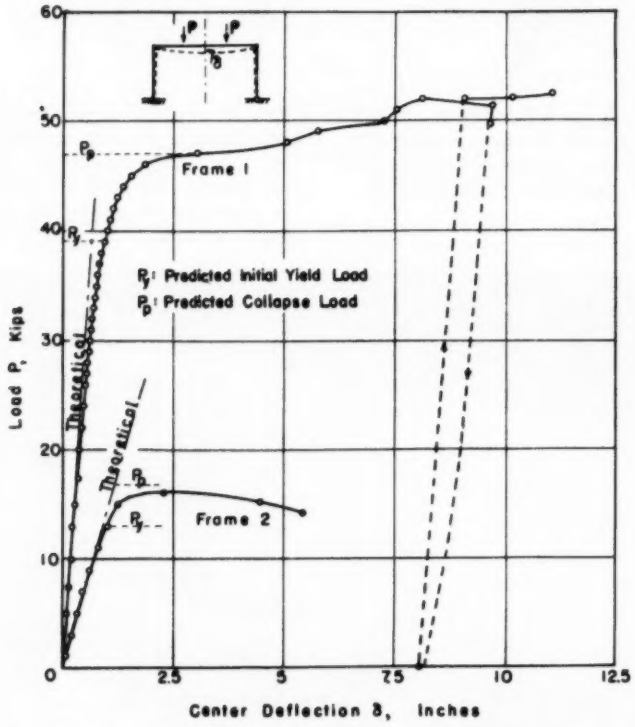


Fig. 14. Load deflection curves for tests of two portal frames.

In one frame, composed of 8-in. WF40-lb. sections, when the mid-span deflection had reached about 10 percent of the span-length, the frame was still taking increasing loads. Due to strain hardening, this frame carried loads considerably in excess of the collapse load that had been predicted by plastic analysis. Large unit rotations were developed in this 8WF40 section.

The second frame, composed of 8-in. light beam, wide-flange sections, 8B13, almost developed the predicted collapse strength, despite the fact that it is one of the least suitable shapes available to resist local buckling. In the test of a third frame, made since the original presentation of this paper, the effect of combined lateral and vertical loading was studied. The results conformed very closely to those predicted by plastic analysis.

In a test of an 8-in. WF @ 31-lb. column 16 ft. long, with hinged ends and an axial load and bending moment applied at the top, the maximum resistance of the column not only exceeded the predicted yield moment but exceeded the plastic hinge moment for this beam section subjected to pure bending. This indicates that in many cases the influence of axial load may be neglected in portal frame columns when designing by plastic analysis.

From the various tests of columns under this program, a good deal of fundamental information has been derived. It will be used in developing rational methods for designing columns, including the interaction or combined effect of bending and axial loading.

Tests were made of beams continuous over two supports, with loaded overhanging cantilever arms, to simulate a portal frame, prior to the tests of actual frames. This arrangement afforded flexibility in regulating the slope of the beam over the supports to simulate various conditions of behavior.

Four different tests were made of such continuous beams, as delivered, with rolling mill residual stresses and some localized residual welding stresses at the supports. In the early part of the plastic range, there was about a 10 percent reduction in moment capacity as compared with calculated values. This has been attributed mainly to the effect of rolling mill residual stresses. However, with continued straining, the full plastic hinge moment can be developed. Even a 14WF30 beam, lightest of its series and therefore tending to buckle locally, showed a fairly good combination of plastic strength and rotation capacity.

Of the three types of stiffening details used over the supports for the continuous beams, Fig. 15, type C was the best, resulting in the least deflection, and type A was the poorest detail.

In the continuous beam tests, the effect of residual stresses and stress concentrations seemed to be only a moderate increase in deflections. Substantially the predicted ultimate strengths were developed in all tests. A test made of an 8WF67 beam after stress-relief annealing, showed moment-rotation characteristics conforming very closely to theoretical values.

The behavior of the beams in these various tests, together with other considerations, lead to a belief that residual stresses due to welding are not usually as influential as those due to cooling after rolling. The welding stresses are highly localized whereas the rolling mill stresses are widespread and would be expected to cause somewhat premature yielding over a wider area and thus influence deflections.

The portal frame tests, and the tests of columns under combined bending and axial loads of the order to be expected in the legs of portal frames, all indicated little if any effect of axial loads upon load capacity. Therefore, residual stresses would not likely be an important factor. In members subjected to relatively high axial loads, they might exert more influence, especially in the case of a pin-ended column.

This method of plastic analysis has been thought of mainly as applying to frames of buildings. Before it could be applied to bridge design, there would have to be an investigation of the effect of small plastic deformations upon fatigue strength.

In any event, to apply the method widely and to the fullest extent of its benefits, further studies would be desirable to determine the local buckling characteristics of the various rolled sections that are available for use in design. Local buckling of the edges of beam-flanges in the plastic range was the immediate cause of collapse for nearly all of the specimens that were loaded to their ultimate capacity. One of the objectives of the Lehigh program

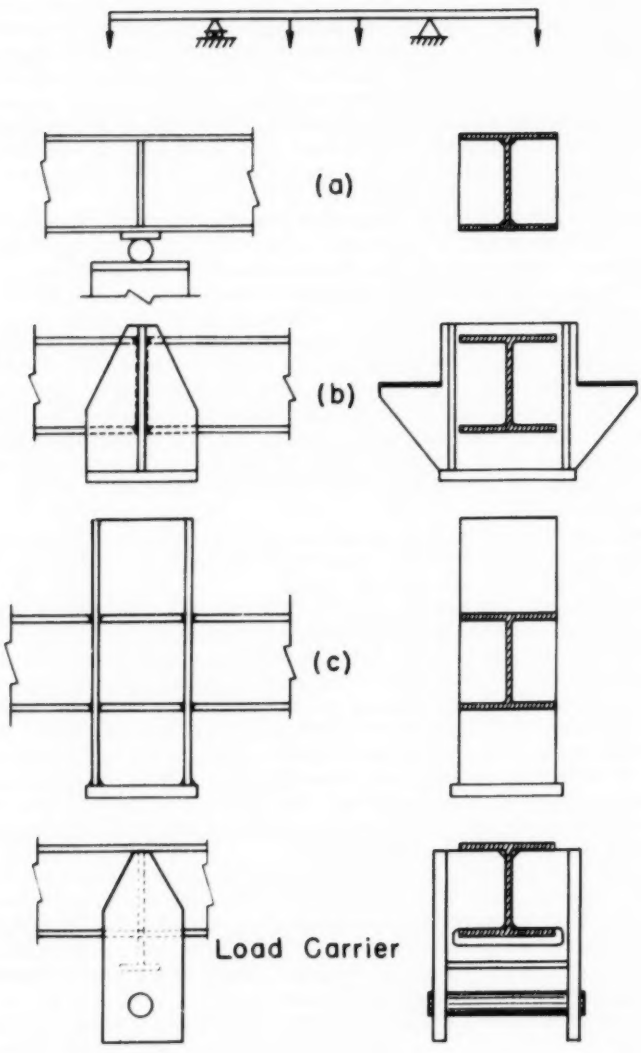


Fig. 15. Details of load carrier and of three types of supports used in tests of continuous beams with loaded cantilever arms.

is to develop a specification for the most suitable geometrical shape or cross section for rolled steel sections.

At least one structure has been designed by the method of plastic analysis, a building for the British Welding Research Association.

As more thought is being given to resistance to bombing, the matter of reserve strength becomes increasingly important. Whether it is recognized or not in design analysis, experience confirms that welded rigid frame and continuous construction usually provides a tremendous amount of reserve strength as compared with simple beam and column design.

Canadian Welded Bridge Difficulties

All together some 15 large welded girder bridges have been constructed in the Province of Quebec, Canada. With the exception of one bridge at Three Rivers (Fig. 16), the most recently constructed of these welded bridges, they have been giving trouble free service for years. The Three Rivers bridge is composed of two groups of continuous plate girder spans, totaling some 2000 feet in length, and composed of span-lengths up to 180 feet. Some time after this bridge had been put in service, two fractures developed in one of the girders, about three years ago, when the temperature dropped quite rapidly from +40 deg. F. to 10 deg. below zero. Repairs were made and rivited cover straps were added at all welded tension joints similar to those where fractures had occurred, because an investigation had shown some of the $2\frac{1}{2}$-in.-thick flange plates at the fractures to be of an inferior rimmed steel of very high average carbon and sulfur content, containing marked segregations. This steel is very much different from that ordinarily furnished for bridge construction in the United States and Canada.

The following winter, on January 31st, 1951, at 3:00 A.M., in 30 degree below zero weather, four spans of this continuous girder bridge collapsed completely. A very thorough investigation of the cause of failure is still underway. The collapsed structure could not be removed and examined for quite a while. Design, fabrication and erection operations, welding methods, procedures and sequences, have all been investigated thoroughly by an independent consultant, and given a "clean bill of health."

An examination of the steel after removal from the stream, revealed innumerable fractures throughout the members; but none of them occurred at welded joints or splices in the flanges or webs. The findings of the judicial Commission set up to determine the cause of collapse was inconclusive, attributing the failure to an "unknown scientific cause" or to "sabotage." A demolition expert who examined the steel after collapse, testified that the local appearance at one point and general conditions indicated that collapse had been caused by an explosion.

Of course, they necessarily had to traverse longitudinal fillet welds in some cases. The girders were composed of single thickness flanges fillet welded to the webs.

A report on the investigation following the first fractures was presented in January, 1951, by D. B. Armstrong, Chief Engineer of the Dominion Bridge Company, at an Annual Meeting of the Highway Research Board in Washington, D. C., and it was published as noted in the list of references at the end of the paper.

The shortcomings of the steel material, resulting in very poor low-

Fig. 16. Construction of Three Rivers Bridge in Province of Quebec, Canada.

Fig. 17. Welded Canadian Bridge at Ste. Anne de la Parade has withstood severe low winter temperatures for 16 years. Photo by courtesy of the Dominion Bridge Company.

temperature notch toughness, may well be considered as the cause for the very brittle behavior as the spans collapsed. However, the primary cause for collapse has not yet been definitized. It does not appear that there was anything faulty in the design or the welding methods that can be held accountable, in view of the locations of the fractures during the collapse, especially when riveted straps had been added at critical points during the earlier repairs.

A plausible hypothesis has been advanced by disinterested parties, to account for the forces causing initiation of such a failure with virtually no live load on the bridge. This hypothesis is that non-thermal expansion or growth of the concrete floor slab provided such forces. From photos of the partial fractures that occurred in 1950, it appears that the steel had parted about $\frac{3}{4}$ in., but the adjacent concrete slab had not. This seems to indicate high compression stress in the floor slab.

There is particularly strong adherence to this theory by bridge engineers who have had experience with this kind of expansive concrete in the Middle West, where certain kinds of aggregates have been used. Such concrete has been found in various parts of the United States and Canada, some of it in locations quite near to the Three Rivers bridge.

The floor slab of this bridge was quite well anchored to the steel girders and floor beams, although composite action was not assumed in the design calculations. A thorough investigation of the concrete is being carried out, with particular emphasis upon the non-thermal expansion or growth phenomenon. The results should throw further light upon the interaction of stresses that existed between the girder flanges and concrete deck of this particular bridge.

George W. Lamb, M. ASCE District Engineer of the A.I.S.C., at Topeka, Kansas, and formerly Bridge Engineer of the Kansas Highway Commission, has made computations which indicate that the amount of non-thermal expansion that might well have occurred in the floor slab of this bridge, could easily cause fracture and failure under these circumstances. He is a capable stress analyst and a bridge engineer of long practical experience, and is very familiar with the behavior of such expansive concrete.

Concrete of this kind has grown as much as 2 in. per 100 feet in some bridge floors in Kansas. With no anchorage to continuous steel beams, the floor has slid along them and sheared off back walls of abutments. In a bridge composed of a series of simple spans, since the steel was not continuous, it moved along with the concrete, displacing the superstructure with respect to pier bents. The greater part of the growth of such concrete has been found to occur during the first 2 to 5 years.

Whether expansive concrete is deemed to be a major contributing factor, or not, in the case of the Three Rivers Bridge, it is well for designers to be aware of this phenomenon, especially if they contemplate taking advantage of composite action or tying concrete floor slabs into steel girders for any other reason, in comparatively long structures. Under such circumstances it is especially important to make suitable tests to insure the furnishing of materials that will not produce this expansive type of concrete. Such tests have now been developed.

As evidence of the adequacy of other welded steel bridges in Canada, to withstand their severe low winter temperatures, the Ste. Anne de la Perade Bridge is a typical example (Fig. 17). Fabricated and erected by the same Dominion Bridge company, it has been serving faithfully through winter after winter, since its construction some 16 years ago.

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